DEVELOPMENT AND PRACTICAL APPLICATION OF A HEALTH MONITORING SYSTEM FOR SHORT- AND MEDIUM-SPAN BRIDGES BASED ON PUBLIC BUS VIBRATION

Ayaho Miyamoto[1]

ABSTRACT : Bridge health monitoring using information technology and sensors is capable of providing more accurate knowledge of bridge performance than traditional strategies. In this paper, a new condition assessment method of existing short- and medium-span reinforced/prestressed concrete bridges was proposed based on vibration monitoring data obtained from a public bus. This paper not only describes details of a prototype monitoring system that uses information technology and sensors capable of providing more accurate knowledge of bridge performance than conventional ways but also shows a few specific examples of bridge condition assessment based on vehicle vibrations measured by using an in-service public bus equipped with vibration measurement instrumentation. This paper also describes a sensitivity analysis of deteriorating bridges based on simulation of the acceleration response of buses conducted by the "substructure method" employing a finite element model to verify the above bridge performance results. The main conclusions obtained in this study can be summarized as follows: (1) Because the vibration responses of passenger vehicles, such as buses, have a good linear relationship with the vibration responses of the target bridges, the proposed system can be used as a practical monitoring system for bridge condition assessment. (2) The results of sensitivity analysis performed by the substructure method show that bus vibration responses are useful for evaluating target bridge performance. (3) The proposed method was applied to a network of real bridges in a local area to evaluate its effectiveness. The results indicate that the proposed method can be used to prioritize the repair/strengthening works of existing bridges based on various vibration information in order to help bridge administrators establish rational maintenance strategies.

KEYWORDS: short- & medium-span bridge, public bus, condition assessment, vibration, SHM, field test, practical application, characteristic deflection, information technology

1. INTRODUCTION

In view of the fact that many of the bridges in Japan are short- or medium-span (10~20 m) bridges, this study aims to propose a rational and feasible method of assessing declines in safety performance due to structural damage, paying attention to the “acceleration” and “deterioration” stages in deterioration process of existing bridges, which are particularly important from the viewpoint of safety management. In this study, as one solution to the problem, a new monitoring method using a public bus as part of a public transit system (*Bus monitoring system*; see Figure 1) is proposed, along with safety indices, namely, characteristic deflection, which is relatively free from the influence of dynamic disturbances due to such factors as the roughness of the road surface, and a structural anomaly parameter. A basic study was conducted by using the results of technical verification experiments and numerical analysis simulation. In this study, attention is paid only to changes in structural characteristics, which are basic performance attributes of a bridge. Local deterioration and factors contributing to such deterioration, therefore, are not identified. It has been shown that the effects of structural anomalies of bridges and measured physical quantities show certain tendencies [1]. If a bus is used, it is important to distinguish changes in information caused by structural anomalies from other changes. Changes in this category may include changes in the road surface profile and changes in structural characteristics of the bus [2]. The characteristic deflection and structural anomaly parameter

[1] Graduate School of Science & Engineering, Yamaguchi University, Japan, Professor, Dr. Eng.

proposed in this study are indices that are relatively free from the influence of external disturbances caused by the roughness and type of the road surface, as long as the structural conditions of the bus remain unchanged.

The other aim of this paper is to introduce the validation experiment for a medium- and short-span bridge monitoring system which is newly developed to evaluate bridge conditions through vibration measurement conducted by using a publicly operated fixed-route bus, and describes the validation results and various findings related to the practical use of the system in the coming years. An acceleration sensor is installed under the rear spring of a fixed-route bus, and characteristic deflection is calculated from the vibration measured when the bus passes over a bridge. If the vibration thus measured reaches a certain level, it is judged that the bridge is in a serious condition of one kind or another. In such cases, a detailed visual inspection is conducted as soon as possible, and appropriate repair or strengthening measures are taken. In the validation experiment conducted in Ube City, Yamaguchi Prefecture, Japan, a sensor was installed to an in-service fixed-route bus, and a total of 39 measurements were conducted over a period of about one year from December, 2010. As the first step, the bridges existing on the routes of the fixed-route bus service operated by Ube City were counted, and bridges to be monitored and a fixed-route bus to be used were selected. Comparison of the measured under-rear-wheel-spring acceleration response of the bus and the measured midspan-of-girder acceleration response of each bridge revealed that when passing over a target bridge, the passing bus causes the bridge to vibrate induced. Vehicle-induced vibration simulation by the “substructure” method [3], which is a finite element method, was also conducted for the selected bridges to determine deterioration evaluation criteria. In the vehicle-induced vibration simulation, differences in calculated values of characteristic deflection and the structural abnormality parameter between different analysis models were evaluated. Finally, the degrees of abnormality of the monitored bridges were expressed with the "inspection necessity level" to enable local governments to prioritize their bridge maintenance needs quantitatively.

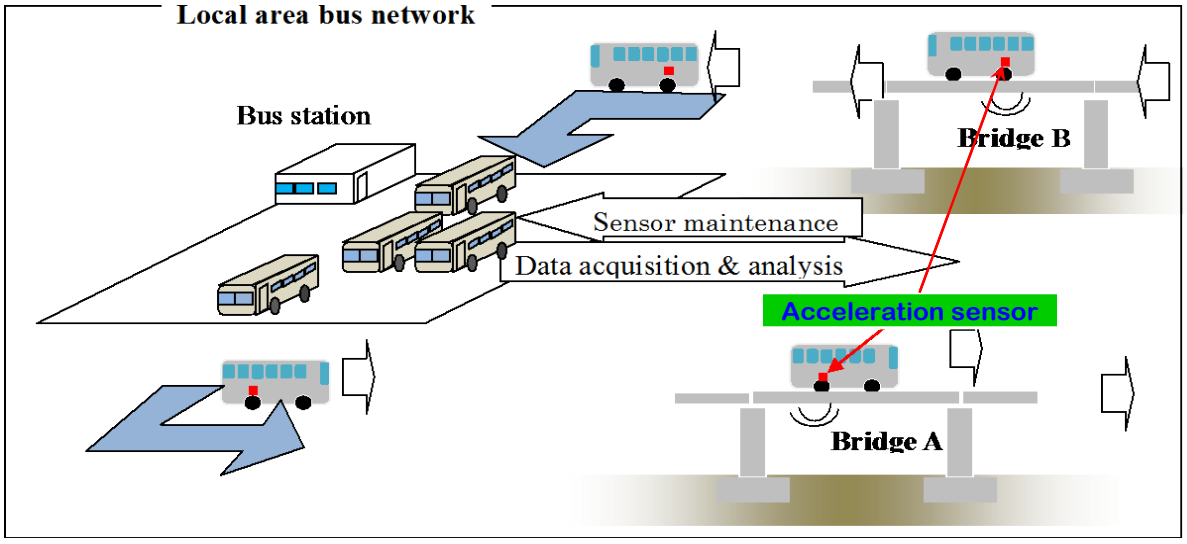


Figure 1. Concept of bus monitoring system for short- and medium-span bridges

2. BACKGROUND AND AN IDEA OF BUS MONITORING SYSTEM

Structures in the final phase of the “acceleration” stage or an early phase of the “deterioration” stage in deterioration curve are at high risk of showing a sudden change in an anomaly, as shown in Figure 2, because of the relationship between cumulative damage probability due to performance declines and changes over time. It is possible, therefore, that a sudden change in an anomaly that could cause fatal structural damage over time is overlooked. If such anomalies are not detected and left unattended, a serious management problem might result. An effective way to prevent such problems is to conduct

inspections at shorter intervals, but it is difficult to do this in reality because of the difficulty in retaining specialists and economic limitations. In order to detect anomalies in a bridge at the final phase of the “acceleration” stage or an early phase of the “deterioration” stage, therefore, it is necessary to use a method that is simpler and easier to use than visual inspection. An effective means of doing so is continuous monitoring by use of Structural Health Monitoring (SHM) sensors directly installed on bridges. SHM has currently received an increasing attention in the field of civil infrastructure systems in many countries. However there are delicate discussions and applications still needed to be improved.

A maintenance approach dependent solely on bridge maintenance based mainly on visual inspection or the SHM method, which requires installing sensors on each bridge, is not suitable for maintaining a large number of deteriorating bridges, since there are many problems to be solved in order to achieve long-term continuous monitoring while meeting the technical and economical rationality requirements. Research results related to onboard monitoring systems include the detection of anomalies of railway tracks [4,5] and the assessment of the soundness of road structures by use of a multifunctional inspection vehicle [6]. The author has also found many useful research results concerning dynamic bridge interactions, such as simulation-based studies on impact coefficients [7,8,9]. Many other useful research results have also been reported regarding basic theories and experimental studies on bridge–vehicle interactions during the passage of a vehicle in connection with bridge health assessment [10].

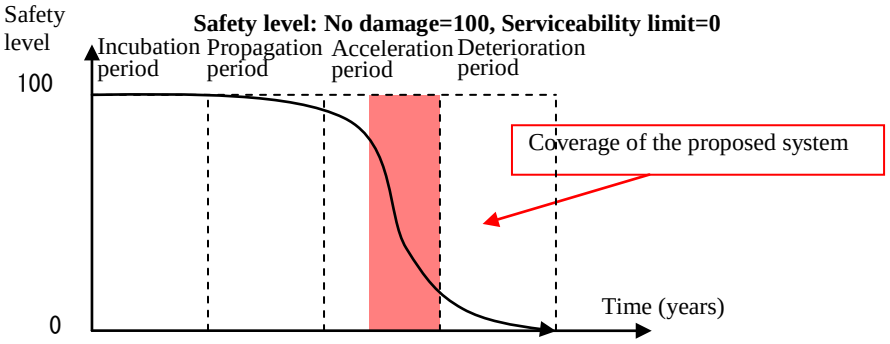


Figure 2. Reduction in safety level and coverage of proposed system

3. THEORETICAL APPROACH OF THE SYSTEM BASED ON BUS VIBRATION

When a vehicle passes over a bridge, its mechanics model can be expressed as a dynamic interaction between the equation of motion of the bridge, expressed by Eq. (1), and the equation of motion of the vehicle, expressed by Eq. (2). The method of formulating a bridge structure model and a vehicle structure model as separate equations of motion and expressing their interaction at joints with input and output vectors is called the "substructure method [3]".

$$\mathbf{M_m\ddot{u}_m + C_m\dot{u}_m + K_m u_m} = \begin{Bmatrix} \mathbf{R_s} \\ \mathbf{R_m} \end{Bmatrix} \tag{1}$$

$$\mathbf{M_s\ddot{u}_s + C_s\dot{u}_s + K_s} \begin{Bmatrix} u_{s_f} \\ u_{s_g} \end{Bmatrix} = \begin{Bmatrix} 0 \\ \mathbf{R_s} \end{Bmatrix} \tag{2}$$

where, the parameters are as follows:

$\mathbf{M_m C_m K_m}$, $\mathbf{M_s C_s K_s}$: mass/damping/stiffness matrix for bridge and for vehicle,

$\ddot{u}_m \dot{u}_m u_m$, $\ddot{u}_s \dot{u}_s u_{s_f}$: response acceleration/velocity/displacement vector for bridge and for vehicle,

u_{s_g}: input forced displacement vector for vehicle, and

$\mathbf{R_m R_s}$: support reaction vectors for bridge and vehicle.

For bridge–vehicle interactions, the vehicle wheel reaction to the bridge is input as the load vector $\mathbf{R_s}$,

and the bridge deflection $\delta(t) \in u_m$, and road surface unevenness $\lambda(t)$ are input to the vehicle as the force displacement vector u_{s_g}. Changes in bridge stiffness K_m are reflected in the measurement results for both the bridge system and the vehicle system. Then, it can be concluded that, because structural anomalies of the bridge due to deterioration, etc. are reflected in changes in the nodal response $\ddot{u}_s \dot{u}_s u_{s_f}$ of the vehicle system, anomalies of the bridge, in theory, can be detected from the vehicle.

Here, let us consider substructure segmentation based on the bus body–wheel–bridge concept, as shown in Figure 3. First, let us take a look at the bus body–wheel system. In a mechanics problem in which the difference method approximation holds true, when forced displacement including the bridge deformation $\delta(t)$ is input to the wheel of the bus, the equation of motion can be approximated, as shown in Eq. (3), by using the proportionality constant P, which is dependent on physical quantities that remain unchanged in the system, such as time, stiffness, damping and mass, and the state constant Co (known) before time obtained by a Taylor expansion, etc [11]. This means that the system's response to the input vector is allocated proportionately depending on the system-dependent constants.

$$u_{s_f} \cdot P(M_s, C_s, K_s, t) + Co(M_s, C_s, K_s, t) = F(t) \tag{3}$$

where, the parameters are as follows:

$M_s C_s K_s$: mass/damping/stiffness matrix for the vehicle,

u_{s_f}: displacement vector,

Co: known value before time t obtained by Taylor expansion, etc.,

$F(t)$: input vector to vehicle system, and

P: proportionality constant dependent on system.

Next, let us consider wheel–bridge vibration transmitted from the bus body. The response of the wheel and the bridge to the input from the bus body is allocated proportionately depending on the physical constants of the system. Let A_b represent the response vector of the bridge, and A_s, the wheel response vector of the bus. Then, we have matrix P expressed as shown below:

$$A_b = P^{-1} A_s P \tag{4}$$

This means that if Eq. (4) holds true in a mechanics problem in which the difference method approximation holds true, then the vibration behavior of the bridge is proportional to the wheel vibration of the bus, and changes in A_b due to bridge anomalies are reflected proportionately in changes in wheel vibration, A_s, of the bus.

Next, let us explain the rationale behind the method for extracting damage/deterioration-related information from measured rear wheel vibrations of a bus without being affected by the dynamic characteristics of the bridge and the vehicle or the road surface profile. The rear wheel vertical vibration $\delta_a(t)$ of a bus running at a constant speed can be expressed as the sum of static displacement $\delta_{sa}(t)$, which is dependent on the measured rear wheel vertical vibration of the bus, and dynamic displacement $\delta_{da}(t)$, which is dependent on the road surface profile and the vibration characteristics of the bridge and the vehicle:

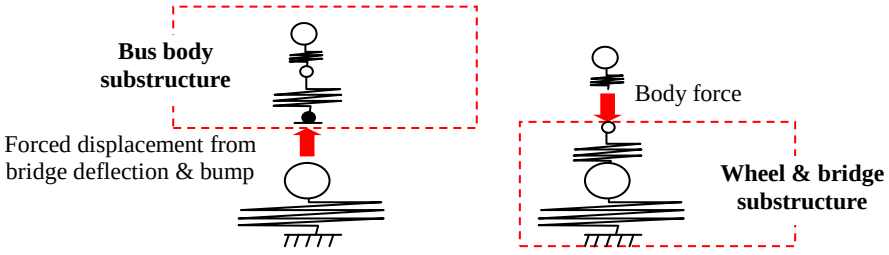


Figure 3. Substructures for modeling bridge and bus body

$$\delta_a(t) = \delta_{sa}(t) + \delta_{da}(t) \tag{5}$$

If it is assumed that the unevenness of the road surface can be expressed as a steady-state random Gaussian process with an average value of 0 and that dynamic displacement including bridge–vehicle interaction is ergodic and Fourier-series-expandable, the dynamic displacement $\delta_{da}(t)$ can be expressed as their sum:

$$\delta_{da}(t) = S_r(\Omega, t) + \int_{-\infty}^{+\infty} X(f) \cdot e^{2\pi f t i} df \tag{6}$$

where, the parameters are as follows:
$S_r(\Omega, t)$: density function for unevenness of road surface,
Ω: spatial frequency of road surface profile, and
$X(f)$: Fourier coefficient.

In Eq. (6), the sample mean in the second term is 0. The mean of a sufficient number (N) of samples obtained from $\delta_{da}(t)$ can be expressed as,

$$\frac{\sum_{t=1}^{N} \delta_{da}(t)}{N} \fallingdotseq 0 \tag{7}$$

Because the sample mean follows a normal distribution in accordance with the central limit theorem, $\delta a\,(t)$ on verges to a certain value (μ_a). For the sample mean $\delta_a(j)$ of a sufficient number (n) of samples, sampling is performed so that there is no overlap in the samples. If N is sufficiently large for $nk = N$ and $t = 1 \sim N$, the following equation holds true:

$$\mu_a = \frac{\sum_{i=1}^{n} \overline{\delta a\,(t)}_i}{n} = \frac{\sum_{i=1}^{nk} (\delta_a(j))_i}{nk} \fallingdotseq \frac{\sum_{t=1}^{N} \delta_a(j)_t}{N} = \frac{\sum_{t=1}^{N} \delta_{sa}(t)}{N} \tag{8}$$

This means that the mean displacement of the rear axle of the bus passing over the bridge obtained from a sufficient number (N) of samples can be extracted as a value, μ_a (referred to as *characteristic deflection*), that is relatively free from the influence of the vibration characteristics of the bridge and the vehicle and the dynamic displacement due to the unevenness of the road surface. On the basis of a similar assumption, the mean of the deflection $\delta_b(t)$ at a given point on the bridge during the passage of the vehicle converges to a certain value (μ_b) that is relatively free from the influence of dynamic deflection. By using static deflection $\delta_{sb}(t)$, μ_b can expressed as $\mu_b \fallingdotseq \frac{\sum_{t=1}^{N} \delta_{sb}(t)}{N}$. If similarity holds true between $\delta_{sa}(t)$ and $\delta_{sb}(t)$ as discussed earlier, and values extracted from measurements taken after the occurrence of deterioration or damage are represented by μ_a' and μ_b', then the following equation can be defined:

$$\alpha = \frac{\mu_b'}{\mu_b} = \frac{\mu_a'}{\mu_a} \tag{9}$$

The rate of change α expressed by Eq. (9) is defined as the *structural anomaly parameter*. After a given value of α is set, the characteristic deflection is monitored continuously. When the characteristic deflection has exceeded a certain limit, it can be judged that the bridge of interest has moved from the acceleration stage on to the deterioration stage (see Figure 2).

4. FEASIBILITY STUDY OF BUS MONITORING SYSTEM

4.1 Experimental method using a real bridge and a public bus

In order to verify the hypothesis of the similarity between the vertical bus-wheel vibration and the vertical bridge vibration shown in Eq. (4), a verification experiment using a real bridge and a public bus was conducted. Repeatability of measurements and statistical characteristics of the data were also verified to evaluate the feasibility of the proposed monitoring system. The bridge (referred to in this study as the "KW Bridge"), shown in Figure 4, spans the Hinuma River in Ibaraki Prefecture, and the bus has a vehicle weight of 95.5 kN. The single-span reinforced concrete bridge has a span of about 22 m and has four girders and five 0.21-meter-wide cross beams spaced 5.15 m apart. The bridge is 44 years old but has no major damage that would affect its structural performance. Figure 5 shows a view of the underside (girders) of the bridge and the bus. The acceleration sensors were installed at the midspan location of an external girder directly under the path of the bus that passed over the bridge. The experiment was conducted three times each at vehicle speeds of 30 km/h and 40 km/h. The vehicle speed was controlled by visually monitoring the speedometer.

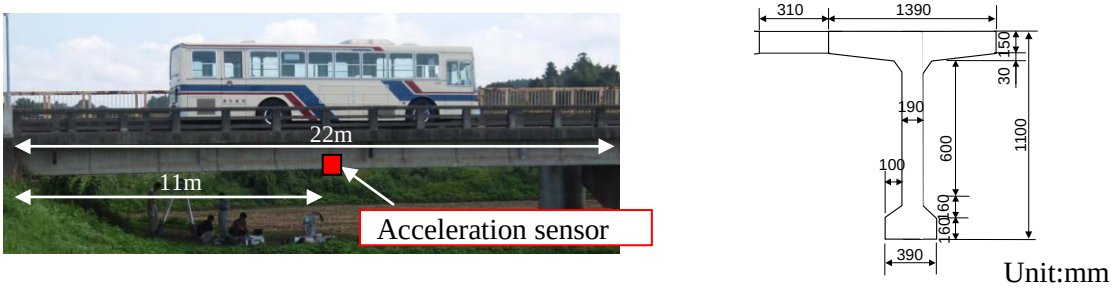


Figure 4. Dimensions of target bridge (KW-Bridge) and sensor locations

Figure 5. Details of main girders (bottom view) and sensor location on a girder and the bus

4.2 Experimental results and discussion

Figure 6 shows a time history of the bridge (midspan) and bus rear wheel response acceleration at 40 km/h, when the bus passed the midspan ("Passing midspan"). At that time, roughly similar vibrations were observed. Similar results were obtained in the others measurements. The distribution of vibration frequency components was determined by Gabor's continuous wavelet transform method. Figure 7 shows a scalogram showing the midspan and bus rear wheel acceleration measurement data obtained at 40 km/h. These experimental results indicate the similarity between the vertical vibration acceleration of the bridge and the rear wheel vibration acceleration of the passing bus. In a mechanics problem with finite differences, the results indicate the high likelihood that the vertical vibration acceleration of the bridge will be approximated by the rear wheel vibration acceleration of the passing bus on the basis of the assumptions indicated in this study.

Figures. 8 (a) and (b) show a time history and the probability distribution of the rear wheel vibration acceleration response obtained from three sets of measurement results obtained when the bus passed the midspan zone at 40 km/h. Measurement errors were balanced out by averaging the results obtained from a number of measurement sessions. This indicates that the vibration of the bridge can be

estimated from data obtained by averaging the results of a number of measurement sessions, and structural characteristics of the bridge can be identified.

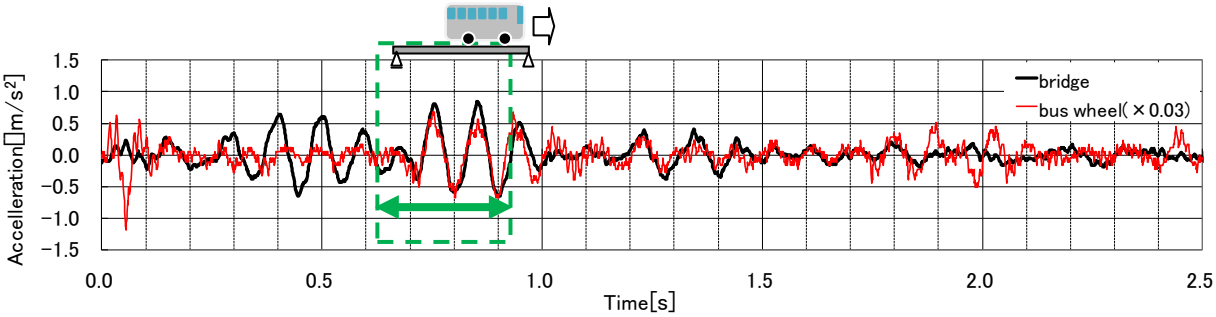


Figure 6. Comparison of bridge acceleration (midspan) and bus wheel acceleration at 40 km/h

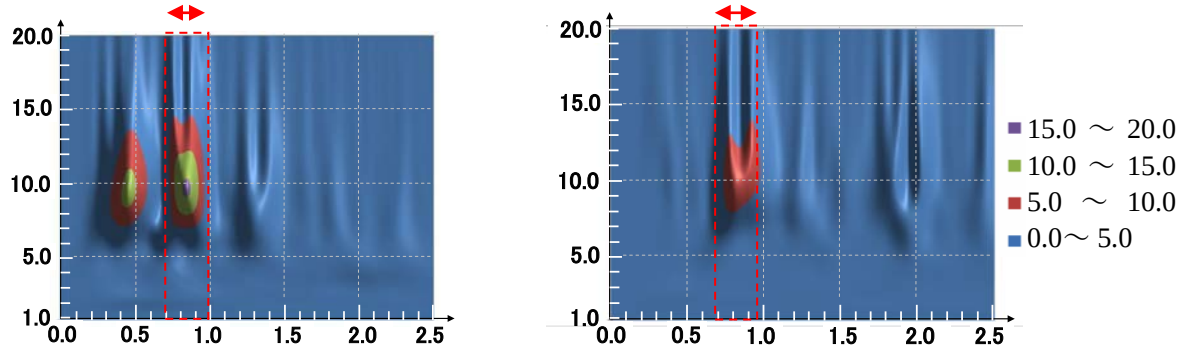
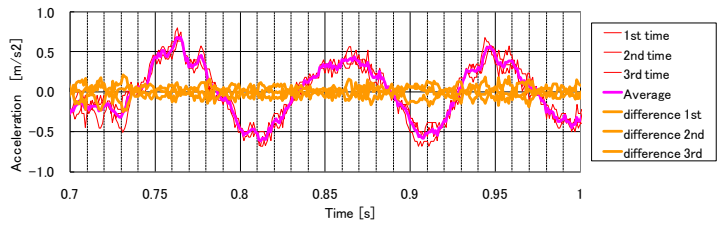
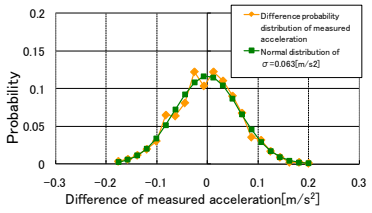


Figure 7. Comparison of Gabor wavelet transform scalogram between bridge girder (midspan) and bus wheel



(a)Time histories



(b) Difference probability distribution

Figure 8. Differences among three measurements of bus wheel (rear axle) acceleration at 40 km/h

5. VEHICLE-INDUCED VIBRATION SIMULATION FOR THE SYSTEM

Figure 9 illustrates the finite element method (FEM), and Table 1 details its material specifications. For the road surface unevenness (bump) parameters, the average parameter values for asphalt pavements were used [12].

On the basis of the predetermined bus body dominant frequency (1.8 Hz) and the wheel dominant frequency (around 10.0 Hz) and the data indicated on the automobile inspection certificate, a four-degree-of-freedom spring–mass model having dominant frequencies roughly matching the predetermined bus body and wheel frequencies was constructed. Figure 10 shows the natural frequencies and mode shapes of the bridge model. Figure 11 shows the natural frequencies and mode shapes of the bus. The body vertical vibration modes and front and rear wheel vibration modes of the bus showed fair agreement with the experimental results (body frequency around 1.8 Hz, wheel frequency around 10 Hz). The measurement and simulation results for the midspan strain when the bus was moving at 30 km/h were compared in order to verify the validity of the analysis model. Strain was

measured with a long-gauge optical fiber sensor. The comparison results are shown in Figure 12. As shown, the simulation results showed slightly greater strains. Judging from the timing of the occurrence of the maximum strain and the tendency of increase or decrease in strain, however, it was thought that the degree of this tendency was low enough to ignore when comparing damage levels (damaged or not).

By referring to reported cases of damage to existing reinforced concrete bridges, two cases of damage at the final phase of the acceleration stage or an early phase of the deterioration stage were assumed: (1) end-to-end spalling at the lower flanges of all girders (Type A damage), and (2) mid-span flexural yielding of one of the girders directly under the lane in which the bus passes (Type B damage).

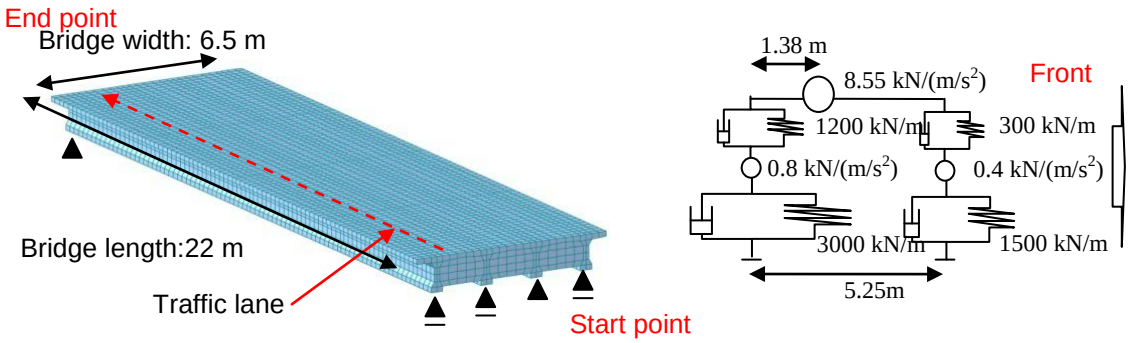
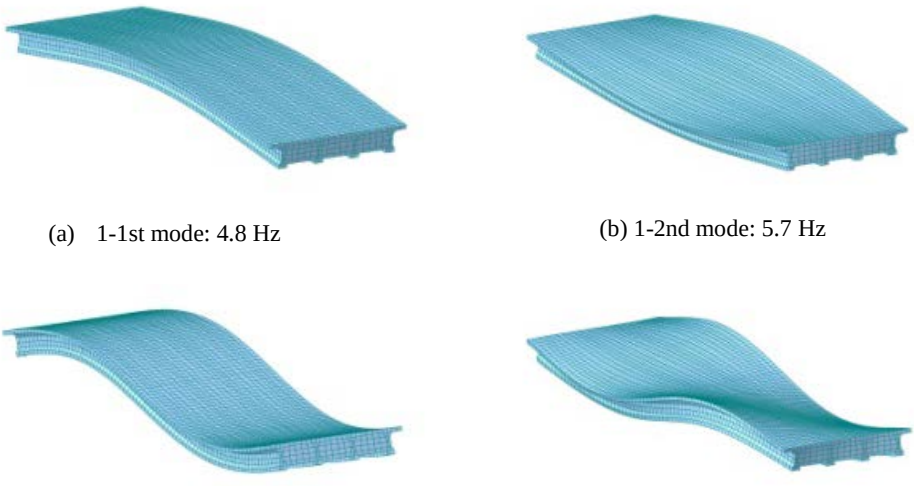


Figure 9. Dimensions and cross-section of KW-Bridge and its finite-element model and the four-degree-of-freedom analytical model of the bus

Table 1. Input data for finite-element analysis

Material properties	Characteristic values
Young's modulus	23.5 kN/mm^2
Poisson's ratio	0.167
Specific gravity	2.5×10^{-8} kN/mm^3
Boundary conditions	Supporting conditions of 4 main girders: End point : Fixed Start point : Moved with longitudinal spring of 200 kN/mm



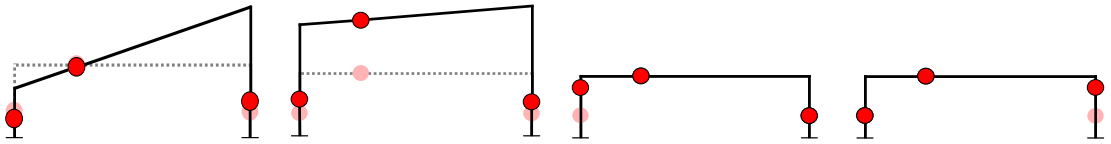
(a) 1-1st mode: 4.8 Hz

(b) 1-2nd mode: 5.7 Hz

(c) 2-1st mode: 13.0 Hz

(d) 2-2nd mode: 15.9 Hz

Figure 10. Target orders and modes of bridge vibration (eigen analysis)



(a) 1st mode: 1.8 Hz (pitching mode) (b) 2nd mode: 2.8 H (body vertical mode) (c) 3rd mode: 10.7 H (front wheel vertical) (d) 4th mode: 11.6 Hz (rear wheel vertical)

Figure 11. Target orders and modes of bus vibration (eigen analysis)

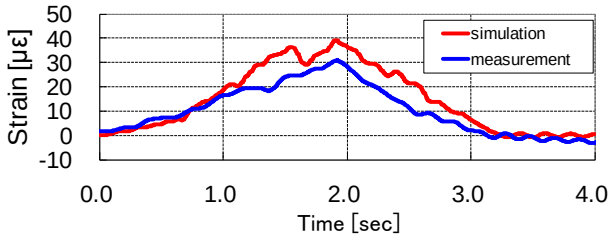
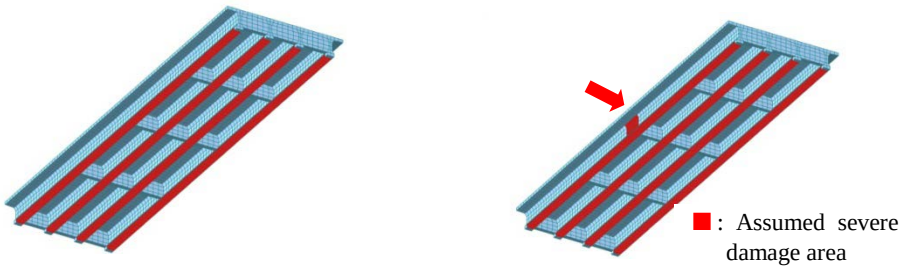
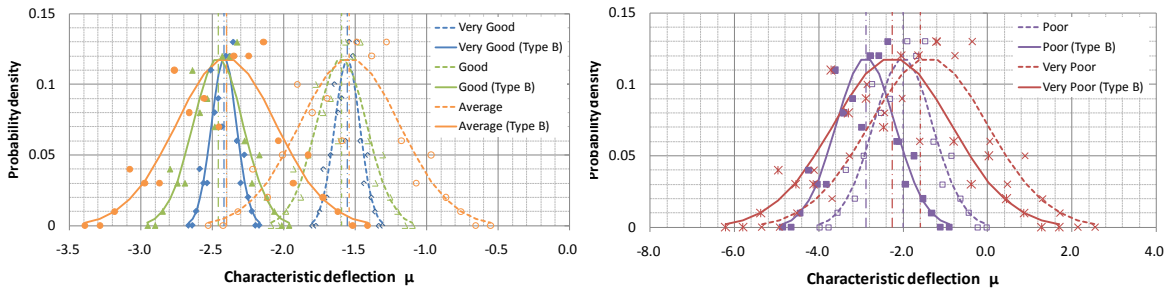


Figure 12. Comparison of strain behavior between measurements at midspan lower flange of main girder and simulation results obtained by FEM (30 km/h running speed)



(a) Type A severe damage (b) Type B severe damage

Figure 13. Damage type and location of assumed severe damage area



(a) Very Good- Average (b) Poor- Very Poor

Figure 14. Distribution of characteristic deflection probability for Type B severe damage

The assumed damage locations are shown in Figure 13. According to the effective mass ratios in the vertical direction obtained from the eigenvalue analyses, the effective mass ratio in the 1-1st vibration mode was dominant (77%). Therefore, by applying the concept of a plastic hinge often used in seismic design, etc., to the mid-span region located near the peak of the mode shape, the stiffness of the plastic hinge region was reduced to 1/100 of the normal value by the effect of damage due to flexural cracking, etc. The vehicle speeds assumed were 30 km/h and 40 km/h, as in the experiment. For the road surface condition (roughness), the five-level criteria defined by the ISO (from "very good" to "very poor") for road surface roughness evaluation was used.

For each evaluation criterion, 100 different patterns of road surface profile were generated by Monte Carlo simulation. Under these conditions, simulation was conducted for a total of 3,000 cases. Figure 14 shows a distribution of the characteristic deflection for different road surface profiles.
Examination of the differences in the distribution of the characteristic values depending on the existence or nonexistence of damage reveals that deterioration-induced changes are overwhelmed by surface irregularities if the road surface is in a "poor" or better condition. The examination also reveals, however, that if the road surface is kept in a condition better than "average," deterioration-induced changes can be identified with relative clarity, regardless of surface irregularities and vehicle speed.
Table 2 shows the average values of characteristic deflection for different vehicle speeds and road surface profiles in the no-damage and damage (Type A and Type B) cases. As shown in the table, the structural anomaly parameter α is 1.40 in the case of Type A damage and 1.55 in the case of Type B damage.

Table 2. Average values of characteristic deflection (midspan)

Speed and road condition	Not damaged μ a	Type A severe damage μ a' (A)	Type B severe damage μ a' (B)
30 km/h, Very good	-1.55	-2.17	-2.41
30 km/h, Good	-1.57	-2.19	-2.43
30 km/h, Average	-1.53	-2.15	-2.39
40 km/h, Very good	-1.56	-2.18	-2.42
40 km/h, Good	-1.60	-2.22	-2.46
40 km/h, Average	-1.54	-2.16	-2.40
Average	-1.56	-2.18	-2.42

Vertical displacement was estimated by using acceleration wave integrals obtained, by the Fourier transform method, from the rear wheel vertical vibration acceleration of the bus passing over the bridge in the experiment in order to calculate the characteristic deflection. The average value of the characteristic deflection in the no-damage cases was –0.4464. Table 3 shows the characteristic deflection values. If, on the basis of the simulation described earlier, Type A damage (α = 1.40) is used as a monitoring criterion, it can be judged that some kind of serious structural anomaly has occurred if the average value of the characteristic deflection is lower than –0.625 mm.

Table 3. Average values of characteristic deflection in KW-Bridge experiment

Speed	μ_a (no-damaged)		
30 km/h	-0.4568	-0.5327	-0.4524
40 km/h	-0.3078	-0.3793	-0.5495
Average	-0.4464		

6. PRACTICAL APPLICATION OF THE SYSTEM TO BUS NETWORK

6.1 Bus monitoring test in Ube City

In the verification test, an acceleration sensor was installed under the rear wheel spring of an in-service fixed-route bus (Figure 15), and the deflection of each bridge when the bus passed over it was estimated from the acceleration response of the bridge. The specifications of the bus and acceleration sensor used in the experiment are summarized in Tables 4(a) and (b). The acceleration sensor installed under the rear wheel spring of the bus is shown in Figure 15. The acceleration sensor was bonded to the underside of the rear wheel spring and coated with epoxy resin for protection. The cables from the acceleration sensor were routed through a drain hole in the floor of the bus cabin. Figure 15 shows also

how acceleration was measured. A portable battery was used in the experiment although the acceleration sensor, data logger and the computer of the monitoring system are supposed to be powered by the battery of the bus. In the experiment, in order to investigate the influence of operating conditions such as weather, the number of oncoming vehicles, vehicle speed and the number of occupants on characteristic deflection, measurement on the bus was conducted by two persons. Of the two persons, one, seated at the back of the bus, recorded the time at which the bus passed over each bridge and the number of occupants, and the other, seated at the front of the bus, recorded the number of oncoming vehicles and bus speed.

The verification test was conducted over a period of one year from December, 2010, with the cooperation of the Civil Engineering and Construction Department and the Department of Transportation of Ube City. Table 5 summarizes the bridge information on some city bus routes of the verification tests. The bridges to be monitored and bus routes were selected so that the monitoring bus passes over the “Jyase Bridge”, which has more spans (5 spans) than any other bridge on the bus routes, as often as possible. This study focused on the “Jyase Bridge”, “Shiratsuchi Daini Bridge” and the “Shingondai Bridge” so as to increase the number of measurement passes as much as possible. Information on the three bridges is summarized in Table 6.

Table 4. Bus and acceleration sensor specifications

(a) Bus

Item	Specifications
Net vehicle weight	8,130kg
Gross vehicle weight	11,485kg
Front/front axle weight	2,730kg
Rear/rear axle weight	5,400kg
Distance between front and rear wheels	4.4m

(b) Acceleration sensor

Item	Specifications
Acceleration sensor	Fuji Ceramics SA11ZSC-TI
Type of sensor	Piezoelectric type
Sampling rate	1000HZ

Figure 15. Measuring equipment of the bus monitoring system

Table 5. Measuring conditions on the verification tests

Item	Description
Period	December 1, 2010 to November 11, 2011
Measurement period	39 days
Bus routes	Nishi Ube–Koto Line Nishigaoka–Nisseki Line Obayama Line Chuo Byoin Line
Bridges	12. Nameless bridge 33. Myojin Bridge 39. Nameless bridge 52. Miyagawa Bridge 53. Myojin Bridge 6490. Jyase Bridge 6570. Shiratsuchi Daini Bridge 8609. Shingondai Bridge

Table 6. Detailed data on monitored bridges

Bridge ID	Bridge name	Date completed	Type of superstructure			Span(m)	Bridge length(m)
386490	Jase Bridge	1976	Span number	Start point 1	PC slab bridge; pretensioned slab	18.0	85.0
				2	PC slab bridge; pretensioned slab	16.0	
				3	PC slab bridge; pretensioned slab	18.0	
				4	PC slab bridge; pretensioned slab	14.0	
				End point 5	PC slab bridge; pretensioned slab	19.0	
386570	Shiratsuchi Daini Bridge	1933	Span number	Start point 1	RC girder; T-girder	7.0	15.8
				End point 5	RC girder; T-girder	7.0	
388609	Shingondai Bridge	June, 1998	Girder bridge/others			22.4	23.6

Figure 16 shows an example of comparison of the under-rear-wheel-spring acceleration response of the bus and the girder midspan acceleration response, both of which were measured when the bus passed over the bridge. As shown, the two waveforms show a fair degree of similarity although the under-rear-wheel-spring acceleration response tends to be greater. From this, it can be concluded that the vibration behavior of a bridge can be accurately extracted from the under-rear-wheel-spring vibration of the bus [13,14].

Based on the above mentioned both acceleration responses, it is necessary, as the first step, to extract data obtained when the bus was passing the midspan zone of the bridge in order to determine characteristic deflection. Figure 17 shows an example of a method of extracting such data for the

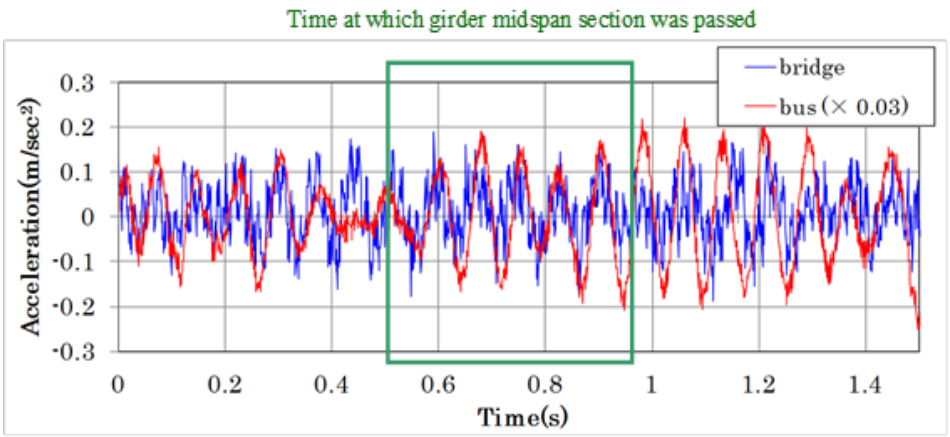


Figure 16. Comparison of under-rear-wheel-spring acceleration response and girder midspan acceleration response

"Shingondai Bridge". In the case of the 23.6-meter-long "Shingondai Bridge", data corresponding to an about-10-meter-long midspan section were extracted, and deflection was estimated by integrating acceleration twice.

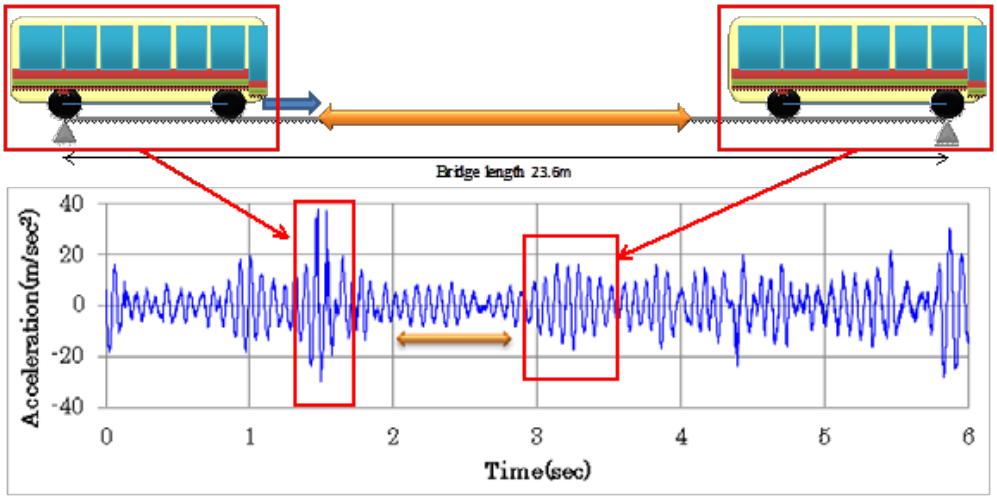


Figure 17. Example of data extraction section (in case of Shingondai Bridge)

6.2 Test results and discussions

(1) Effects of bus operating conditions on characteristic deflection

Figure 18 shows examples of the estimated deflections from bus vibration during the process in which the bus passed over the "Shingondai Bridge" in the direction from the "Nishikiwa Gakkomae" bus stop toward the "Tokonami" bus stop (N→T) (initial trip) at the different seasons. The data extraction

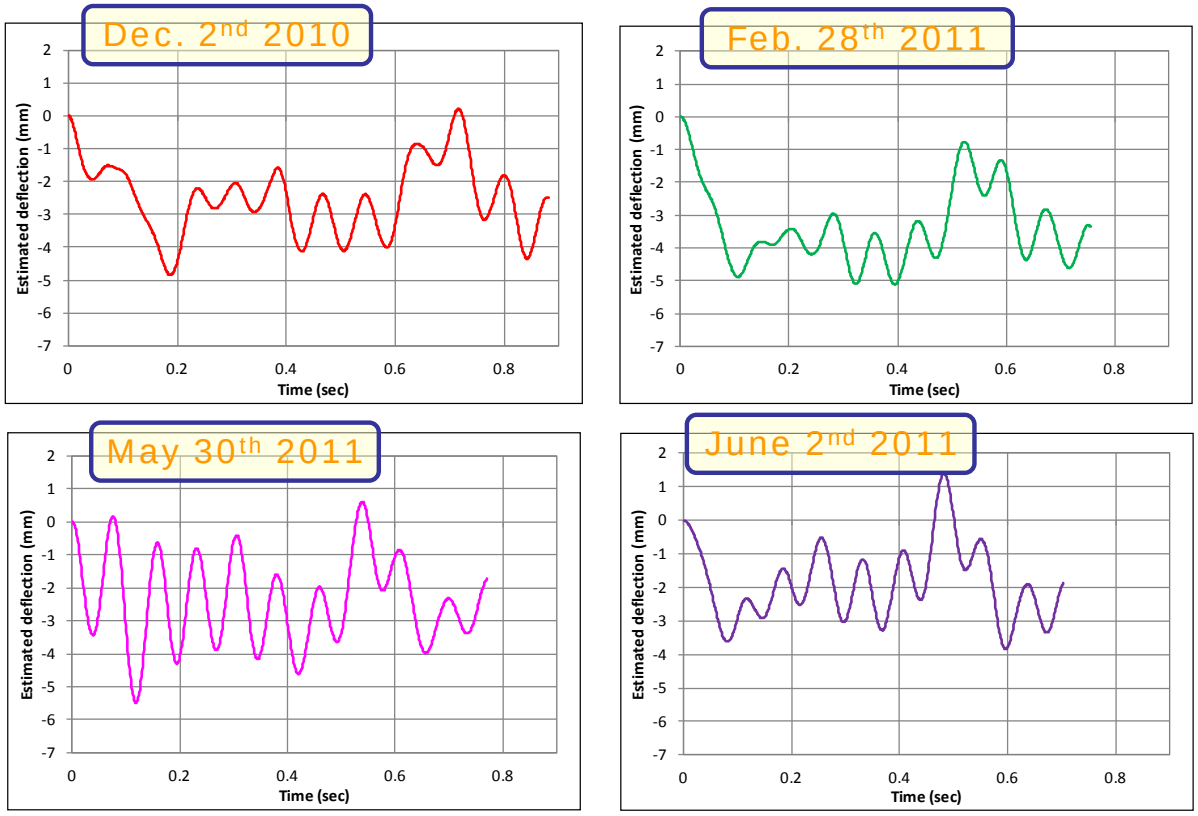


Figure 18. Example of estimated deflection from bus monitoring system

section was determined so that similar waveforms can be obtained from all acceleration responses. The reason why data length varies is that the bus speed was not constant during measurement. Based on the estimated deflections as shown in Figure 18, Table 7 shows an example of measured characteristic deflections calculated by time-averaging the estimated deflections. From this results, it can be inferred that although numerical values vary by the influence of bus operating conditions on the characteristic deflection, calculated values can be made to converge by acquiring a large amount of data and taking averages.

Table 7. Example of measured characteristic deflection with bus operating conditions (Shingondai bridge)

Date	Characteristic deflection	Weather	No. of oncoming vehicles	Bus speed (km/h)	No. of occupants
Dec.1, 2010	-2.13	Clear	0	45	8
Dec.2, 2010	-2.51	Cloudy	1	40	7
Feb.28, 2011	-3.36	Cloudy	1	45	6
May.30, 2011	-2.39	Clear	0	40	12
June.1, 2011	-2.07	Clear	0	45～50	9
June.2, 2011	-1.89	Clear	0	45	13

(2) Data management

Moving averages were calculated by varying the data interval, and standard deviations of characteristic deflections were determined. The results are shown in Figure 19. The term "data interval" refers to the number of data sets from which moving averages are calculated. As shown, as the data interval is made larger, the variability of characteristic deflection values decreases. It can further be seen that after data interval exceeds a certain limit, the standard deviation no longer changes. This means that variability is reduced by averaging characteristic deflections, irrespective of bridges, spans or the direction of travel. In this study, changes in characteristic deflection are evaluated in terms of moving average by using a data interval of 14.

(3) Index of bridge evaluation for monitoring data

Vehicle-induced Vibration Simulation: In here, characteristic deflection corresponding to different states of deterioration is determined through vehicle-induced vibration simulation using the substructure method [3] of finite element analysis. By using the characteristic deflection thus obtained, its rate of change, α, is calculated as a structural abnormality parameter [1]. In this study, three types of models were created for the “Shingondai Bridge”, and structural abnormality parameter values were compared by using those models. The modeling patterns for the “Shingondai Bridge” are shown in Figure 20. The parameters of the bridge model thus prepared are shown in Table 8 [15]. As the simplest model, a simple beam model was considered. In addition to the simple beam model, a plane model incorporating the characteristics of the bridge of interest (Type 1), and a more detailed plane model (Type 2) were also created. By using the simulation results obtained from those models, deterioration evaluation criteria were determined.
Figure 21(a) to (c) show examples of estimated under-rear-wheel-spring displacements (deflection) obtained in the case in which the geometrical moment of inertia was varied (deterioration) under the following simulation conditions: (1) analysis model: Plane Model Type 1, (2) bus speed: 30 km/h and (3) road roughness: average [9]. As shown, estimated displacement tends to increase with the progress of deterioration. It is also evident that the estimated under-rear-wheel-spring displacement of the bus is greatly affected by the roughness of the road surface.
Bridge evaluation index: The calculation of the structural abnormality parameter by use of the abovementioned vehicle-induced vibration simulation results has shown that in the cases of deterioration causing prestressing force to decrease by 50% and 90%, characteristic deflection increases by factors of 1.93 and 2.86, respectively. In this study, an attempt was made at using the structural abnormality parameter thus obtained as a deterioration evaluation criterion in bridge monitoring. Figures 22(a) and (b) show the changes in the characteristic deflection of the “Shingondai

Bridge" calculated in the preceding chapter, along with the deterioration evaluation criteria. The Deterioration Phase 1 and Deterioration Phase 2 criteria used here are the characteristic deflection values that are greater than the initial value by factors of 1.93 and 2.86, respectively.

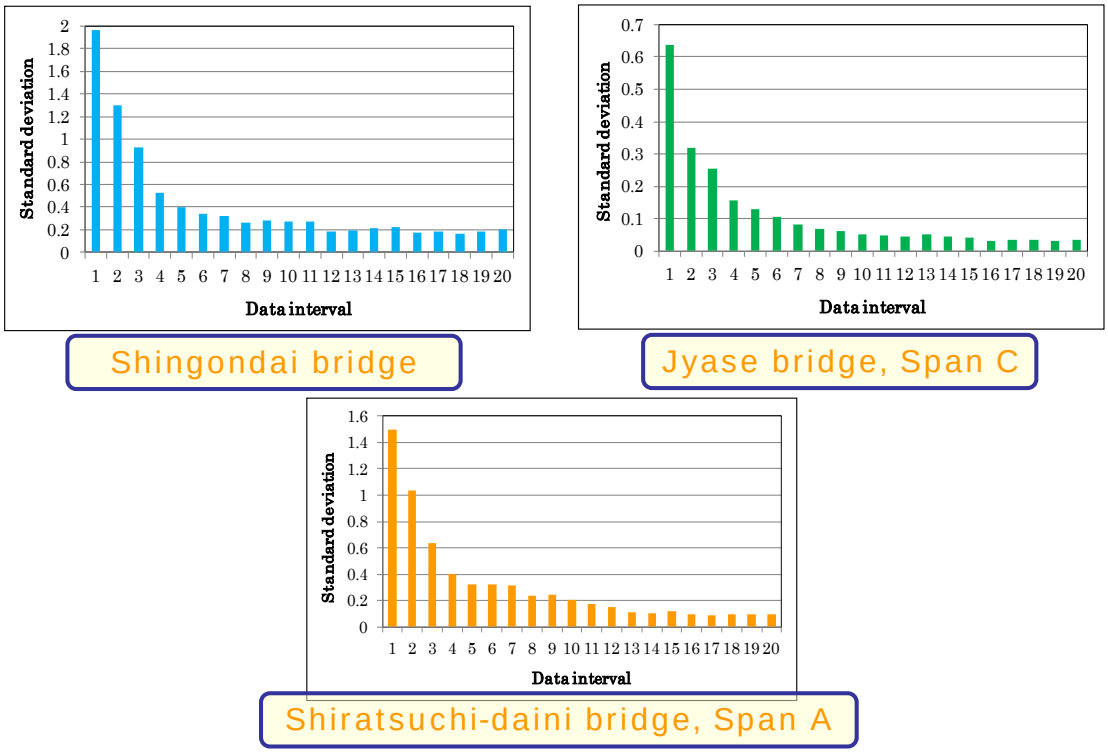


Figure 19. Change of the standard deviation in data management on target bridges

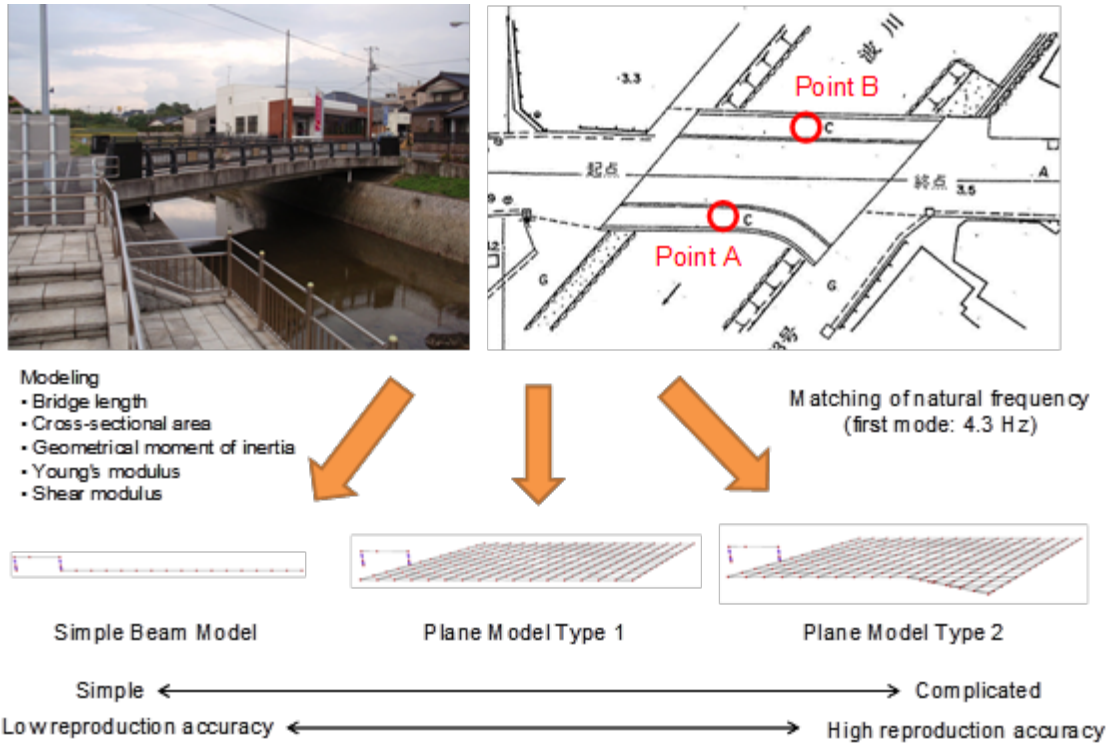


Figure 20. Example of modeling of "Shingondai bridge"

Table 8. Bridge model parameters of "Shingondai bridge"

Parameter	Set value
Cross-sectional area	6.5440m^2
Geometrical moment of inertia	0.3186m^4
Young's modulus	23500N/mm^2
Boundary conditions	Start point: fixed End point: roller

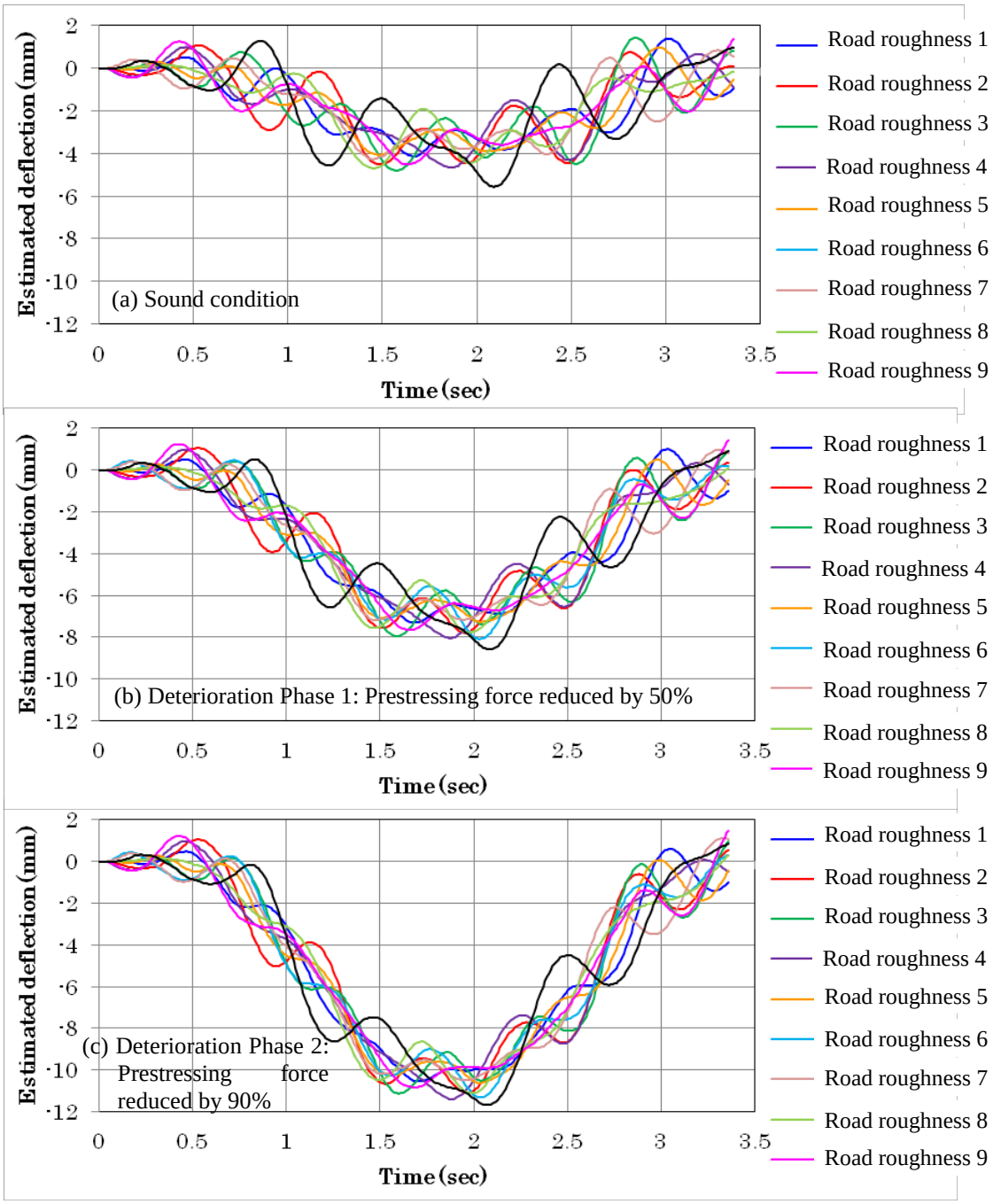
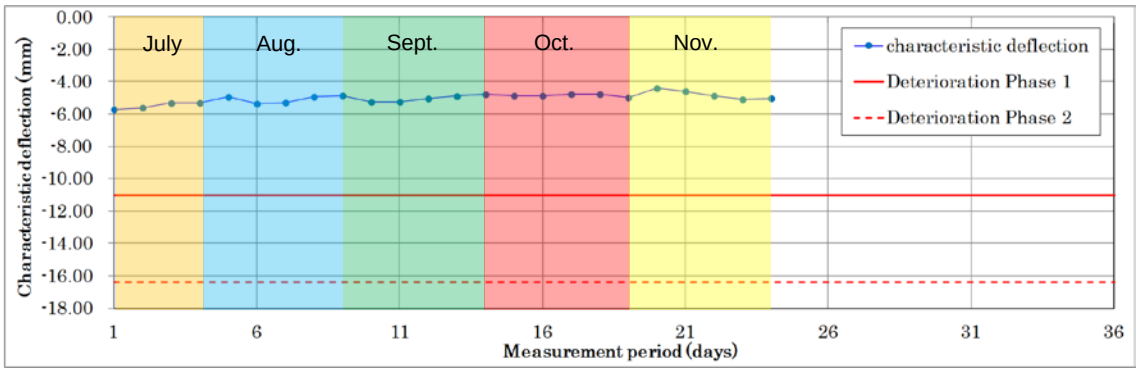


Figure 21. Example of estimated deflection through simulation (Shingondai Bridge, Plane Model Type 1, bus speed: 30 km/h, road roughness: average)

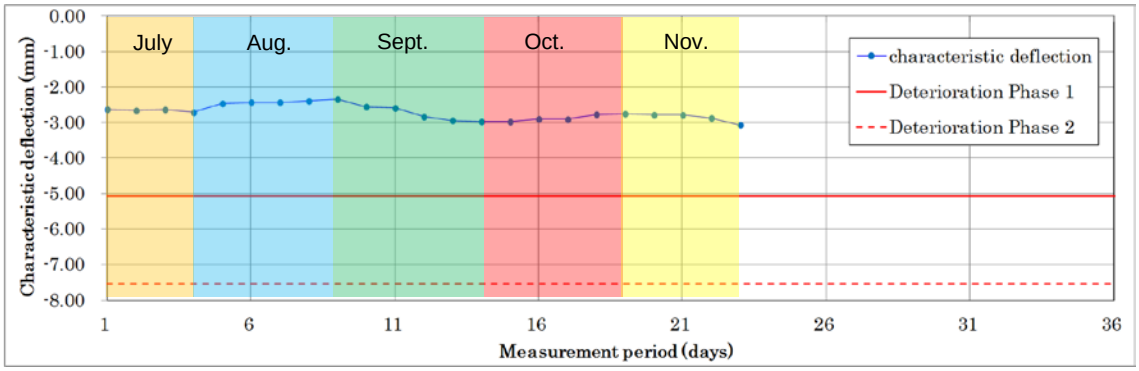
As an indicator for evaluating how close the present state of the bridge under consideration is to the deterioration evaluation threshold and determining whether or not visual inspection is necessary, the "inspection necessity level" is defined. The inspection necessity level indicates how close the lowest value of the measured characteristic deflection is to the Deterioration Phase 1 threshold and is expressed in terms of a percentage calculated as,

$$\frac{\left|\text{Average value of characteristic deflection - Lowest value of past characteristic deflection}\right|}{\left|\text{Average value of characteristic deflection - Evaluation threshold}\right|}\times 100 \qquad (10)$$

Table 9 shows an example of the initial values, averages, standard deviations and lowest values of characteristic deflection of the “Jyase Bridge”, along with the deterioration evaluation criteria. The deterioration evaluation criteria were calculated by using the average value as the basis for calculation. Figure 23(a) and (b) show the results of inspection necessity level calculation by use of Eq. (10). As shown, as in the case of the “Shingondai Bridge”, at present the inspection necessity level of the “Jyase Bridge” is low. It is necessary, however, to keep in mind that with the progress of deterioration, the inspection necessity level covering the range shown in blue will approach 100%.
In deterioration modeling in this study, prestressing force introduced into a prestressed concrete bridge was assumed to decrease in view of the loading test results obtained by the National Institute for Land and Infrastructure Management [16]. In a vilification test [17] conducted in the past, spalling of up to 4 cm from the bottom of the girder was assumed for deterioration modeling of a reinforced concrete (RC) bridge. Thus, deterioration modeling methods are important for the determination of deterioration evaluation criteria. As a next step, therefore, it is necessary to consider various cases.



(a) Tokonami → Nishikiwa Gakkomae



(b) Nishikiwa Gakkomae → Tokonami

Figure 22. Indication of deterioration threshold lines (Shingondai Bridge)

Table 9. Summary of initial value, average value, standard deviation & lowest value of characteristic deflection, and evaluation criteria (Jyase bridge)

			Initial value	Average	Standard deviation	Lowest value	Deterioration Type 1	Deterioration Type 2
Jyase Bridge	Span A	S → K	-1.20	-1.23	0.035	-1.29	-2.41	-3.53
		K → S	-1.41	-1.53	0.085	-1.69	-3.00	-4.12
	Span B	S → K	-1.36	-1.27	0.081	-1.4	-2.08	-3.07
		K → S	-1.60	-1.40	0.079	-1.6	-2.59	-4.34
	Span C	S → K	-1.63	-1.61	0.055	-1.76	-3.45	-5.36
		K → S	-1.15	-0.99	0.048	-1.15	-1.98	-3.45
	Span D	S → K	-1.36	-1.33	0.054	-1.46	-2.35	-3.47
		K → S	-0.99	-0.94	0.070	-1.09	-1.81	-2.84
	Span E	S → K	-1.23	-1.12	0.063	-1.23	-2.27	-3.76
		K → S	-1.41	-1.44	0.099	-1.65	-2.76	-4.02

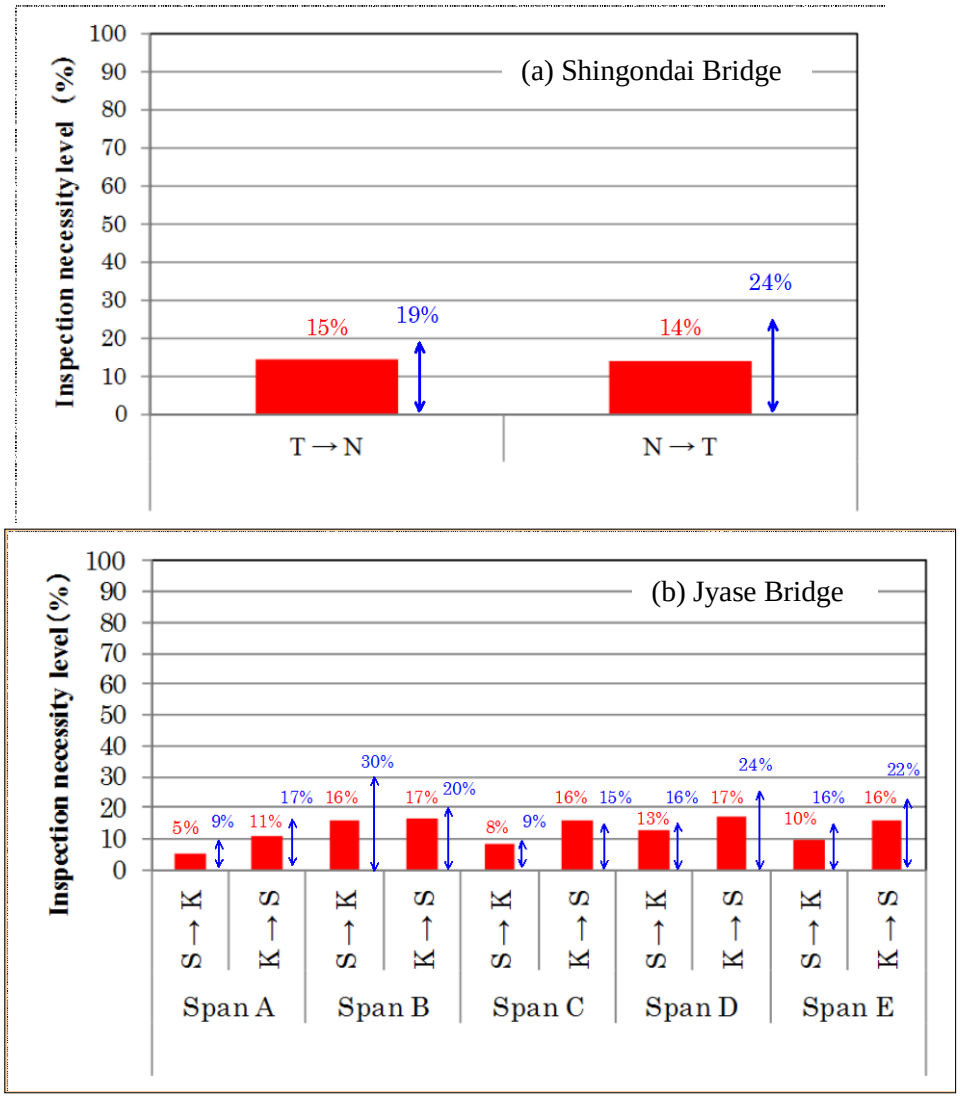


Figure 23. Results of inspection necessity level calculation for "Shingondai bridge" & "Jyase bridge"

7. CONCLUSIONS

In this study, a new method for condition assessment of existing short- and medium-span bridges based on public bus vibration, called "*bus monitoring system*" has been introduced from basic concept to field verification test. In here, it is also discussed about along with safety indices, as a means of solving SHM-related problems, and a fundamental study of the technical problems associated with the use of the proposed method was conducted. And also, this paper has introduced on the results of the experiment carried out for the validation of a proposed bridge monitoring system using an in-service fixed-route bus over a period of one year from December, 2010. Main results reached through analysis and the results of verification experiments and simulation can be summarized as follows:

1) By using the similarity matrix *P* under certain conditions, vertical vibration of a bridge can be estimated from the wheel vibration of a bus passing over the bridge.
2) Structural anomalies of a short- or medium-span bridge with a relatively damage-free ("average") road surface at the acceleration or deterioration stage can be detected by monitoring the characteristic deflection measured from wheel vibrations on the condition that the stiffness of the bus used for measurement remains unchanged.
3) In the event of short-span bridge damage as assumed in this study, an anomaly can be detected from the wheel vibration of the bus used for measurement. A monitoring system for detecting major signs of anomalies occurring around or during the transition from the acceleration stage to the deterioration stage by using a bus is feasible.
4) The method was applied to an actual bus (bridge) networks in Ube City area as a specific example to verify its effectiveness. As the results, it will be able to make a rational long-term health monitoring system for existing short- and medium-span bridges, then the method helps bridge administrators to establish the rational maintenance strategies.
5) Vehicle-induced vibration simulation has shown that although estimates of the under-rear-wheel-spring displacement of the bus is significantly affected by bus speed and road roughness, characteristic deflection is a parameter that is relatively free from the influence of vehicle speed and road roughness.
6) As an indicator for evaluating how close the present state of the bridge under consideration is to the deterioration evaluation threshold and determining whether or not visual inspection is necessary, the "inspection necessity level" has been defined. This has made it possible to quantitatively prioritize bridge management tasks.

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